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Abstract: An experimental study of the shear behavior of recycled aggregate concrete (RAC) beams with and without shear reinforcement is presented. Nine full-scale simply supported beams were loaded in four-point bending tests until failure. Three different replacement ratios of coarse natural with coarse recycled concrete aggregate (0%, 50%, and 100 %), and three different shear reinforcement ratios (0%, 0.14%, and 0.19 %) were the main parameters. All natural aggregate concretes (NAC) and recycled aggregate concretes (RAC) were designed and experimentally verified to have similar compressive strength and workability. It was found that the shear behavior and the shear strength of the beams with 50% and 100% of recycled concrete aggregate was very similar to that of the corresponding natural aggregate concrete beams. The applicability of different code provisions for the shear strength predictions of the RAC beams with and without shear reinforcement was tested by comparison to test results obtained on 85 beams, 58 RAC and 27 corresponding NAC beams. The shear strength of RAC50 and RAC100 beams with and without shear reinforcement was conservatively predicted by the analyzed codes with similar reliability as for the corresponding NAC beams shear strength. At this state-of-knowledge, the application of the analyzed codes' provisions for NAC beams shear strength can be recommended both for the RAC50 and the RAC100 beams.

Highlights

- Nine full-scale NAC and RAC beams were tested until shear failure.
- Similar shear behavior and strength of RAC and NAC beams.
- Codes' provisions for the NAC beams shear strength can be also used for RAC beams.

SHEAR BEHAVIOUR OF RECYCLED AGGREGATE CONCRETE BEAMS WITH AND WITHOUT SHEAR REINFORCEMENT

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Abstract

An experimental study of the shear behaviour of recycled aggregate concrete (RAC) beams with and without shear reinforcement is presented. Nine full-scale simply supported beams were loaded in four-point bending tests until failure. Three different replacement ratios of coarse natural with coarse recycled concrete aggregate (0%, 50% and 100 %), and three different shear reinforcement ratios (0%, 0.14% and 0.19 %) were the main parameters. All natural aggregate concretes (NAC) and recycled aggregate concretes (RAC) were designed and experimentally verified to have similar compressive strength and workability. It was found that the shear behaviour and the shear strength of the beams with 50% and 100% of recycled concrete aggregate were very similar to that of the corresponding natural aggregate concrete beams. The applicability of different code provisions for shear strength predictions of the RAC beams with and without shear reinforcement was tested by comparison with test results obtained on 85 beams, 58 RAC and 27 corresponding NAC beams. The shear strength of RAC50 and RAC100 beams with and without shear reinforcement was conservatively predicted by the analyzed codes with similar reliability as for the corresponding NAC beams shear strength. At this state-of-knowledge, the application of the analyzed codes' provisions for NAC beams shear strength can be recommended both for the RAC50 and the RAC100 beams.

Keywords: recycled aggregate concrete; reinforced concrete beams; experiment; shear strength; codes

1. Introduction

Recycled aggregates produced from demolished concrete, commonly by crushing, screening and removing contaminants by magnetic separation, water cleaning or air-sifting are known as recycled concrete aggregates (RCA). Recycling generally represents a way to convert a waste product into a resource. It has the potential to reduce the amount of waste disposed of in landfills, preserve natural resources, and provide energy and cost savings while limiting environmental disturbance. Consequently, recycling of demolished concrete and the use of RCA in new structures may have key importance in achieving sustainable construction.

RCA is commonly used in lower quality product applications such as back-fills and road sub-base and base, where they compare favourably to natural aggregates (NA) in many local markets today [1]. However, only a small amount of RCA is used today for higher quality product applications such as structural concrete. Although in many countries standards allow the utilization of RCA in structural concrete [2], actual application remains limited to less than 1% of the amount of aggregates used in structural concrete [3]. On the other hand, the potential of demolished concrete recycling to decrease the environmental burdens of concrete can be fully utilized only if RCA replaces NA in structural concrete, since this is by far the largest application of aggregates. Such a concrete in which NA is replaced with RCA (partially or completely) is called recycled aggregate concrete (RAC).

Several experimental studies on the shear behaviour and shear strength of RAC beams have been performed [4–14]. In all of them, full-scale beams in four-point bending tests were loaded until failure, where the following parameters were varied: replacement percentage of NA with RCA, concrete compressive strength, cross-section size, longitudinal and shear reinforcement ratio and shear span-to-depth ratio. The effect of shear reinforcement on shear strength [4–6,8] was less investigated, i.e. there are more test data on the shear strength of RAC beams without shear reinforcement [7,11–14].

From the reported research a few important conclusions are drawn. In the case of RAC beams without shear reinforcement, cracking load, crack patterns, load-deflection behaviour and type of failure are very similar to those of the corresponding natural aggregate concrete (NAC) beams, regardless of the replacement ratio. Premature cracking and lower initial stiffness was observed by some researchers but generally without effect on the shear strength [5,13]. However, non-negligible decrease in failure load of 11% and 16% in RAC beams with 50% and 100% coarse RCA compared with NAC beams was also reported [6]. Shear strength of RAC beams is slightly lower than the shear strength of the corresponding NAC beams, strength decrease being larger with larger replacement ratios. The largest strength reduction (for shear span-to-depth ratios higher than 2) was reported in [12] for a 100% replacement ratio and it was about 25% compared with the corresponding NAC beam. When the equivalent mortar volume mix proportioning method (EMV) is used [15], the shear strength of RAC beams with 63.5 % and 74.3 % replacement ratios is higher than the shear strength of corresponding NAC beams [7].

Comparisons to code predictions in the published research were usually performed only for own experimental results. RAC shear strength was mainly conservatively predicted by different codes, regardless of the replacement ratio. Some researchers concluded that codes for NAC are applicable [5,11,13] while according to others the ratio of test-to-code predicted capacity is higher for corresponding NAC beams than for RAC beams with a 100% replacement percentage [12,14] and therefore the applicability is questionable.

Regarding the RAC beams with shear reinforcement, no significant difference between the shear behaviour and shear strength of RAC and corresponding NAC beams with shear reinforcement was reported. The code predictions are much less consistent and more conservative than in the case of shear strength without shear reinforcement, both for RAC and NAC beams. However, the test database is relatively small [4,6,8]. Furthermore, the application of existing empirical relations for calculating the concrete contribution to the shear resistance of conventional reinforced NAC to RAC beams is questionable because of different findings regarding the aggregate interlock contribution [6,9,10].

Although a lot of research on the mechanical, time-dependent and durability related properties of RAC has been performed in the course of the past few decades [16–18], a comprehensive research on the structural behaviour of RAC is still lacking. This is especially important for the types of structural behaviour that are mainly governed by the properties of concrete, such as shear strength, punching shear strength and short and long-term deformational behaviour of structural elements. Therefore, a comprehensive test database is needed for gaining confidence in the applicability of this material for structural use, as it was done in the past for conventional concrete – NAC with a cement binder.

This paper presents the results of the experimental study on the shear behaviour of RAC beams, obtained in full-scale, four-point bending tests up to failure on nine simply supported beams. Three different replacement ratios of coarse NA with coarse RCA (0%, 50% and 100 %), and three different shear reinforcement ratios (0%, 0.14% and 0.19 %) were the main parameters in this study. All NAC and RAC mixtures were designed and experimentally verified to have similar compressive strength and workability.

2. Objective

The objective of the presented work was twofold. One aim was to widen the existing experimental database on the shear strength, crack pattern and stress distribution of RAC beams with large replacement ratios of coarse NA with coarse RCA (50% and 100%) with and without shear reinforcement. A detailed explanation of RAC beams' shear behavior together with reported test data will contribute to the definition of parameters for finite elements analysis (FEA) of RAC beams and increase the reliability of FEA models.

The other aim was to determine whether code equations for NAC are applicable for the prediction of shear strength of RAC beams, based on the comparison with different codes and including test data currently available in literature. **Furthermore, understanding of shear mechanisms in RAC beams could be used as a certain contribution to the performance based design of RAC beams according to standard provisions for NAC beams [19].**

3. Materials and methods

3.1 Component materials

The origin of RCA used in this study was twofold: (1) from a demolished 40 year-old reinforced concrete frame structure and (2) from laboratory waste concrete samples. In both cases, the properties of the original concrete were unknown, **similar to the industrial practice where RCA is usually obtained from a mix of concrete waste from different sources and of unknown quality.** After the processing of concrete waste in a mobile recycling plant, the obtained coarse aggregate was sieved into three sizes: 4/8 mm, 8/16 mm and 16/31.5 mm. The fine fraction (0/4 mm) was natural sand in all mixtures, while "Morava" river gravel was used as coarse NA in NAC mixtures. **The physical and mechanical properties of all types of aggregates used in the presented research are given in Table 1. All tests were conducted according to the national standards and CEN provisions [20–25]**. Regarding the most important RCA properties, saturated surface dry bulk density varied from 2400 kg/m^3 to 2480 kg/m^3, while water absorption after 24 h varied from 3.8% to 4.6%, depending on the particle size. **This means that the RCA can't satisfy requirements for the best class (class H), according to Japan's standard JIS A 5021 [26] nor the requirements for the best class of RCA according to the classification proposed in [27]. On the other hand, the values of water absorption were above the recommended lower boundary of RCA for structural use [28–31]. Hence, it can be considered as a representative for the class of RCA that could be usually used for structural elements made of RAC. Results of sieving for all aggregate types and sizes are presented on Figure 1. Obviously, they**

fulfil standard requirements for natural aggregate except in the case of fine grains in the 8/16 mm fraction of RCA, which exceeds the limit for a few percentages. However, it was taken into account during the design of the aggregate mix which also had to fulfil standard requirements [32].

A blended cement with mixed additions of slag and limestone (up to 20 %), CEM II 42.5R, was used in all concrete mixtures.

3.2 Concrete mixtures and properties

In order to investigate the shear performance of reinforced RAC beams three types of concrete mixtures were designed – NAC (fine and coarse NA), RAC50 (50% **by mass** of coarse NA replaced by RCA) and RAC100 (100% **by mass** of coarse NA replaced by RCA). There were two design targets – 28 day compressive strength equal to 40 MPa on 150 mm cubic samples and workability (slump) of 8 ± 2 mm, measured 30 min after mixing, for all types of concrete.

This period of 30 min was adopted as the usual period from the moment of concrete production prior to concrete casting in place. During that period water is available to be absorbed by RCA and that would certainly change the concrete properties including workability. In order to avoid the negative effect of high water absorption of RCA on the workability of concrete an additional quantity of water, added water, was carefully determined in accordance with predetermined water absorption of RCA after 30 min, Table 1. The quantity of cement (m_c) and 'free' water (m_v), for a target design strength were derived from the previously determined relationship between the 28 day compressive strength and water-to-cement ratio (w/c) for both NAC and RAC [33]. For that purpose, several NAC, RAC50 and RAC100 mixes of different w/c ratios were designed and tested [34]. Mixture proportions, i.e. quantities of aggregates, were derived by applying the conventional total volume method, Table 2. According to the described methodology, the effective w/c ratio was the same in the case of NAC and RAC50, while 3.3% more cement was needed for RAC100 to provide the same compressive strength. Air content was assumed to be 2% and no water-reducing admixtures were used.

A standard curing procedure [35] was performed for all concrete samples. **This means that concrete was cast in moulds and protected against drying (by means of wet sheets) at a temperature of 20 ± 5˚C. After 24 h, concrete specimens were removed from the moulds and put in water at a temperature of**

20 ± 2˚C until the testing time. Beside these samples, specimens were cast out of each type of concrete and cured under the same conditions as beams and tested for compressive strength at the age of beams testing.

The properties of fresh and hardened NAC and RAC, based on average values for three tested specimens, are presented in Table 3. It can be seen that the compressive strength was within ±10% of the target strength, while slump was within the desired limits for all concrete mixtures. **There are several potential reasons for somewhat higher compressive strength of RAC50 compared with NAC designed with equal effective w/c ratios: good quality of RCA, higher water demand due to the water absorption of RCA and better aggregate packaging. The 'added' water was determined according to RCA water absorption after 30 min. Although the majority of the total water absorption is carried out during the first 30 min (Table 1), it is possible that the effective, real water absorption was somewhat higher, reducing the effective w/c ratio and providing higher compressive strength of RAC compared with NAC. Furthermore, the combination of round particles of natural river aggregate and angular particles of RCA used in the RAC50 mixture could also have led to better packaging of the whole aggregate mixture and consequently to better compressive strength. However, similar results, i.e. higher compressive strength of RAC50 than NAC based on the same design methodology, have been also found in previously conducted experimental research [33].**

Compressive strength of NAC and RAC50 samples cured as beams were higher than those of samples exposed to standard curing conditions, which can be explained by the lower humidity of samples stored with beams at the test date, after seven days of wetting. Although with similar or even lower compressive strength, the modulus of elasticity of NAC was higher compared with RAC while splitting and flexural tensile strengths were almost the same. Similar results were reported by other researchers [6,36–40].

3.3 Description of beam specimens

The experimental investigation was conducted on nine beams organized into three series defined by different shear reinforcement ratios (ρ_w): 0%, 0.14% and 0.19%. Each series consisted of three beams made from different concrete types: NAC, RAC50 and RAC100. In the first series beams had a shear reinforcement ratio ρ_w =0% and were designated as NAC-1, RAC50-1 and RAC100-1, in the second series the beams had ρ_w = 0.14% (minimum shear reinforcement) and were designated as NAC-2, RAC50-2 and RAC100-2 while the beams in the third series had the highest amount of stirrups (ρ_w = 0.19%) and were designated as NAC-3, RAC50-3 and RAC100-3. The minimum shear reinforcement ratio (0.14%) was calculated according to CEN provisions [24] for natural aggregate concrete. All beams had a rectangular cross section 200 mm wide and 300 mm deep, with a total length of 3.5 m. Stirrups were made of plain bars with a yield strength (f_{yw}) of 300 MPa and a tensile strength (f_{tw}) of 430 MPa (Figure 2), while ribbed bars (f_{yl} = 555 MPa, f_{tl} = 621 MPa) were used as longitudinal reinforcement in all tested beams, Table 4. Only one side of the beams was reinforced with the shear reinforcement defined above, while the other side was reinforced with a higher percentage of stirrups. In that way, the failure side was defined in advance. The details of shear and flexural reinforcement for all beam specimens are shown in Figure 3.

3.4 Test set-up and instrumentation

All the beams were simply supported with a span of 3.0 m and subjected to a symmetric two point load system applied at the third points of the beam span, Figure 4. A shear span-to-depth ratio of 4.2 was kept constant for all beam specimens.

The layout of the test set-up and measuring equipment is presented in Figure 4. Strains in stirrups and longitudinal reinforcement were measured by electronic resistance strain gauges (SSG), while vibrating wire strain gauges (VWSG) were used to measure concrete strains. There were a couple of SSGs on each of the four stirrups in the left shear span of beams with minimum shear reinforcement (sections 2, 3, 5 and 6) and on each of the six stirrups in beams with a higher percentage of shear reinforcement (sections 2, 3, 4, 6, 7 and 8), Figure 4. SSGs were also used for longitudinal reinforcement strain measurement and were located in the middle of the beam span and in the middle of the shear span, Figure 4. In these two sections, pairs of VWSGs with a 100 mm base were mounted at the upper surface of each beam. Beam deflections were measured by eight linear potentiometers (LP) placed also in these two sections and additionally in the two supports sections, with two LPs positioned at each face of the beam. Beside this, the strain rosettes were placed in the middle of the shear span where the passing of the critical inclined crack was expected, Figure 4. They were formed from three LPs with an angle of 60º between each and with a base of 150 mm. The beams were tested at the ages of 24 to 30 days, with a force controlled loading system.

4. Results and discussion

A summary of the results is presented in Table 5 where the values of the following parameters are given: load at the first bending crack ($2P_{cr,fl}$), load at first shear crack ($2P_{cr,s}$), ultimate shear load ($2P_u$), service load (adopted as 40% of the ultimate load) deflection (δ_s), normalized shear stress at the first shear crack formation ($v_{cr,s}/f_c^{1/3}$) and normalized ultimate shear stress ($v_u/f_c^{1/3}$). Shear stress (v_s or v_u) was calculated by dividing the shear force ($P_{cr,s}$ or P_u) by the product of effective depth (d) and width (b) of the beams' cross section and afterwards normalized with the cube root of the compressive strength of a cylindrical specimen (f_c).

4.1 Crack pattern and failure mode

In all the beams without shear reinforcement (NAC-1, RAC50-1 and RAC100-1) the first observed cracks were short, vertical cracks due to bending, in the flexural span of the beam. Their width at that moment was about 0.05 mm, for all beams, as the adopted crack width for the thinnest crack observable by the naked eye.

The first shear crack in beam NAC-1 was a short, 0.15 mm wide crack positioned in the middle third of the beam's height. With a relatively low increase of 10% in load afterward, the diagonal crack propagated very quickly upward to the compression side and simultaneously downward to the support zone followed by the spreading of the crack width up to 1.5 mm. A progressive crack widening occurred with an additional load increase of 20% until the failure of the compressive zone. The slope of that crack was about 30°, Figure 5.

The behaviour of beam RAC50-1 after first shear crack formation was quite different. With the load increase, a relatively small crack extension and widening up to 0.05 mm was recorded prior to a sudden and excessive crack opening of more than 10 mm under a load of $2P_u$ = 183.5 kN and concrete crushing in the compression zone. Instead of a pure diagonal crack as in beam NAC-1, the 'S' crack appeared where the crack angle in the middle third was about 45° but only about 15° in other parts connecting the load and support zone, Figure 5. In beam RAC100-1 the increase in load after first shear crack appearance was followed with linking of several cracks into a 4 mm-wide diagonal crack. It showed a very similar crack pattern and crack angle like RAC50-1, Figure 5. **The difference in the shape of the cracks in RAC and NAC beams was dominantly caused by the different angle of the mid part of the crack (in**

the middle third of the beams height) while the other two thirds were formed in an expected manner. The angle of the first crack was determined by the direction of the principle tensile stress. Namely, from the measured strains in the rosette (Figure 4), principle stresses and their directions were calculated. It turned out that the angle of principle tensile stress in beam NAC-1 was 35° while in RAC50 and RAC100 beams it was 44° and 43°, respectively, and they correlated with the crack angles noticed during the experiment, Figure 5.

In general, beams with shear reinforcement showed similar behaviour under loading, Figure 4. As the load increased after first shear crack formation the number of diagonal cracks increased as well as the length of existing cracks. New diagonal cracks were formed by the inclination of existing vertical bending cracks. The well-known diagonal concrete struts were formed between diagonal cracks and some differences in the cracks' slope between NAC and RAC beams can be found. The slope was somewhat greater in NAC beams (35–45°) compared with RAC50 beams (26–31°) and RAC100 beams (30–35°), Figure 5. The failure mode in all six beams, irrespective of concrete type, was a brittle one in the direction of the main diagonal crack and marked with crushing of concrete in the vicinity of the load, Figure 5. The shear cracks in RAC50-2 and RAC100-2 beams were slightly wider compared with those of the NAC-2 beam at the moment of the first shear crack formation (0.1 mm compared to 0.05 mm) and at the load level approaching beam failure (1.0 mm compared to 0.06 mm). In all other load steps, crack widths were similar or slightly smaller in RAC50-2 and RAC100-2 beams than in the companion NAC-2 beam.

4.2 Shear strength and P–δ relations

For beams without shear reinforcement, the ultimate stress v_u is the shear strength of concrete while for shear reinforced beams it is the sum of concrete shear strength and reinforcement contribution. As expected, v_u increases with the increase of shear reinforcement. NAC-2 and NAC-3 beams had 32% and 50% higher ultimate shear stress, respectively, compared with the NAC beam without shear reinforcement (NAC-1). A similar increase was registered for RAC100 beams: 29% and 56% for RAC100-2 and RAC100-3, respectively, compared with the RAC100-1 beam without stirrups. Stirrup contribution in providing overall shear strength was higher in beams with 50% of RCA replacement where the registered increases were 55% and 71% for RAC50-2 and RAC50-3 beams, respectively, compared with RAC50-1, but it was due to the unexpectedly low failure load for RAC50-1 beam and not because of the effect of RCA use in concrete. With the exception of the RAC50-1 beam, the normalized ultimate shear stress of RAC beams and corresponding NAC beams are within 5%, regardless of the percentage of RCA, Table 5. The relations between the normalized shear strength and deflection measured in the middle of the beam span (section D1) and in the middle of the shear span (section D2) are presented in Figure 6. The first significant load decrease was observed after first shear cracks formed leading to stiffness reduction of all three beams without shear reinforcement. In beams without shear reinforcement, an increase in the load bearing capacity afterwards was mainly due to the contribution of aggregate interlock effect of concrete and it was equal for NAC-1 and RAC100-1 beams (33% and 31%, respectively), but limited to only 15% in the case of RAC50-1, Table 5. **Efficiency of aggregate interlock is the function of the roughness of the crack faces. Under the assumption that the old interfacial transition zone (ITZ), i.e. zone between natural aggregate and old cement matrix, is the weakest link in the complex RAC system, cracks will be dominantly formed in it, cracks faces will be smoother and therefore aggregate interlock less effective. This can be avoided by using an equivalent mortar volume method (EMV) in RAC design [9]. But, if the quality of parent concrete was higher than the newly designed RAC it is possible that the cracks form in the new ITZ and because of the angular shape of RCA aggregate (compared to the shape of gravel aggregate), interlock could be even better in RAC. However, this cannot be clearly validated by the results of this experiment. If the aggregate interlock depends only on the amount of natural aggregate it would be less effective in RAC100 than in RAC50 and that was contrary to the presented results.**

In two groups of beams with shear reinforcement, stirrups were successively activated when diagonal cracks intersected each of them. Aggregate interlock was secured in this way and a significant decrease of the beam stiffness was prevented. It seems that the effectiveness of aggregate interlock was not influenced by the content of RCA. However, the result obtained for RAC50-1 where the formation of the first shear crack was followed by a significant deflection increase and collapse of the beam (Figure 6), means that a final conclusion about the efficiency of aggregate interlock in RAC beams will be drawn after the analysis of all collected test data (strains in concrete and stirrups).

Based on the diagrams presented in Figure 6 it can be stated that there is no significant difference in the stiffness of NAC and RAC beams with shear reinforcement before and after shear cracking. Stiffness reduction up to 10% for the RAC50-2 beam compared with NAC-2 prior to shear cracks is evident and could be considered in correlation to the 12.5% lower modulus of elasticity for RAC50 compared with NAC, Table 3. This is also in line with a 10.9% larger deflection (δ_s) of the RAC50-2 beam compared with NAC-2 under the service load level, Table 5. However, it cannot be stated for the remaining two RAC50 and three RAC100 beams (Figure 6, Table 5), which had similar service load deflections as corresponding NAC beams, although with a 10–12.5% lower modulus of elasticity. This behaviour would suggest a lower reduction of the moment of inertia at the whole beam level in RAC compared with NAC beams.

4.3 Concrete strains

The development of concrete strains (ε_c) in the middle of the flexural span (mark '_1') and in the middle of the shear span (mark '_2') in relation to the normalized shear stress is presented in Figure 7. Positive strain signifies compression while negative strain represents tension. There is no significant difference in the concrete strain values between beams with NA and RCA for the same normalized shear stress, Figure 7. The ultimate concrete strain ranges between 3‰ and 4‰ regardless of the concrete type. In the middle of the shear span concrete compressive strains started to decrease at load close to failure, in all beams irrespective of the aggregate type. This behaviour depended on the amount of shear reinforcement and in the beams without shear reinforcement concrete strains even changed from compression to tension. This suggests that with load increase an elbow-shaped strut was developed. The development of such a strut (which in fact deviates to avoid the cracks) is followed by the formation of tension ties exactly in the area where tension strains or a decrease of compressive strains in concrete were measured [41].

Principal strains and principal stresses in concrete calculated from measured strains in the strain rosette are shown in relation to the normalized shear stress, for the beams without shear reinforcement, Figure 8. Only the strains and stresses up to the load when first shear crack was observed by naked eye are presented. **Tensile strains between 0.5‰ and 0.8‰ correspond to the stress level at the first shear crack observation ($v_{cr}/f_c^{1/3}$ in Table 5). They are several times higher than strains calculated from a linear relationship between measured concrete tensile strength ($f_{ct,sp}$) and modulus of elasticity (E_c) (Table 3) which were about 0.11‰ for all three types of concrete. At the load step when the first shear crack was noticed by naked eye, principal tensile stress between 15 MPa and 20 MPa was**

calculated for all beams (Figure 8) and that is, of course, impossible having in mind the concrete tensile strength. Thus, the difference between what we can see during the experiment ('first' shear crack formation) and what we cannot see (internal structural damage) must be emphasized. This internal structural damage obviously occurred in all three beams several load steps prior to 'first' shear crack formation. The formation of internal, micro cracks can be an explanation for the rapid increase of principle strains which occurred three load steps before the crack was noticed at the surface of the beam, i.e. at a load of 100 kN, Figure 8. The value of principle stress and strain at that moment can be roughly estimated from Figure 8 as 2 MPa and 0.1 ‰ respectively. This stress value is in accordance with the tensile strength of concrete, (calculated as 90% of the measured splitting tensile strength of concrete, Table 3), which meant the formation of the shear crack. Thus, the first shear cracks were actually formed before they were observed by naked eye.

Some differences between calculated principle strain lines for NAC and RAC beams starting from that point of diagram can be found. The increase in strains and normalized shear stress was almost the same for NAC and RAC50 beams, which means that the crack formation in the internal structure of the beams was similar. According to Figure 8, in RAC100 beams this effect progressed more slowly, which might mean that the interfacial transition zone (ITZ) was of higher quality. However, at the final stage, when the first crack was noticed at the surface, the principle strains were similar in all three beams, regardless of the quantity of RCA. Such an explanation can be assessed as a hypothesis which should be proved by further detailed investigation on a micro-scale level.

4.4 Strains in reinforcement

The relation between normalized shear stress and strain in the longitudinal reinforcement in the middle of the flexural span (mark M1) and in the middle of the shear span (marks M2, M4 and M5) are presented on Figure 9. Differences between steel strains between groups of beams made of different concretes are within a 10% margin regardless of the RCA amount, with the exception of beam NAC-3 in section M5, Figure 9. In beams without shear reinforcement, the longitudinal reinforcement did not yield at failure, neither in the middle of the flexural nor in the middle of the shear span. In beams with stirrups, strains close to the yielding point (≈2.5‰, based on data from Table 4) in longitudinal reinforcement at failure were recorded in the middle of the flexural span only.

The crack pattern and layout of stirrups in the shear span of beams in group 2 and group 3 are presented in Figures 10 and 11, respectively, while measured strains in those stirrups are shown in Figures 12 and 13, respectively. Until yield strain was reached there were only negligible differences in strain values between NAC and RAC beams. Yield strain of 1.5‰ shown in Figures 12 and 13 was adopted from the stress-strain diagram of stirrups' steel. At failure load, almost all of the stirrups yielded or just started to yield in both NAC and RAC beams. The exceptions are: stirrup in RAC50-2 at section 6, stirrup in NAC-3 at section 2 and stirrup in RAC50-3 at section 8 (but with strains very close to the yield strain). This means that shear carried by stirrups was very similar in NAC and RAC beams regardless of the RCA amount. Consequently, the rest of shear carried by other components of the shear transfer mechanism – aggregate interlock, dowel effect and capacity of the uncracked portion of concrete at the head of the shear crack, was also similar in all beams. It can be assumed that the dowel effect was similar owing to the same amount of longitudinal and transverse reinforcement in all beams coupled with the fact that a splitting crack along the longitudinal reinforcement wasn't observed in any of the beams. The reasonable assumption is that the capacity of the uncracked portion of concrete was also similar since all concretes had similar compressive strength. This leads to a conclusion that aggregate interlock was similarly effective in both NAC and RAC beams, regardless of the RCA amount and amount of transverse reinforcement. However, a different extent of stirrups' yielding was recorded. This can be a consequence of different distances between the position of the strain gauge on the stirrup and the position of the shear crack. If the crack did not pass through the position of the strain gauge (or in its vicinity) the measured strain could differ from the one in gauges directly intersected by the crack. Since very short strain gauges on stirrups were used (5 mm and 10 mm), the probability that the crack had passed through the strain gauge was small and that could have significantly affected the measured stirrups' strains.

Based on the yield strength equal to 300 MPa and the assumption that all stirrups reached the yield strain at failure, it was calculated that about 50% of the failure load was resisted by stirrups in

NAC-2 and RAC-2 beams, and about 70% of the failure load was carried by stirrups in NAC-3 and RAC-3 beams. This is in line with the redistribution of internal shear forces in NAC beams with web reinforcement given in [42].

The increase in load after yielding of stirrups can be resisted by the uncracked concrete at the head of the shear crack until its capacity is exhausted. In NAC-2 and RAC-2 beams yielding of stirrups predominately started at a stress level approximately equal to $0.15f_c^{1/3}$, while the failure load corresponded to a stress level equal to $0.20f_c^{1/3}$. In NAC-3 and RAC-3 beams yielding of stirrups predominately started at a stress level approximately equal to $0.20f_c^{1/3}$, while the failure load corresponded to a stress level equal to $0.22f_c^{1/3}$. Hence, the load increase after stirrups yielding was significant in NAC and RAC beams with minimum shear reinforcement percentage, while in beams with higher amount of shear reinforcement than minimum percentage this increase was about 10% of the load corresponding to the yielding of stirrups.

4.5 Comparison to codes predictions

The applicability of different code provisions for the shear strength predictions of RAC beams without shear reinforcement was tested on 63 beams, 44 RAC and 19 corresponding NAC beams. Beside own results, test data were taken from [7,8,11–14,16] but only for slender beams with shear span-to-depth ratios larger than 2 and for replacement ratios equal to 50% and 100%.

In all but one of the considered experimental studies, the absorption of used RCA ranged from 3.3% to 6% while the oven-dry density varied from 2300 kg/m^3 to 2430 kg/m^3. According to Silva et al. [27], this RCA can be classified into classes AIII and BI. Only RCA used in the experimental study by Choi et al. [11] had a low absorption value of 1.93% and relatively high oven-dry density of 2480 kg/m^3. These properties were obtained by multiple crushing and removing of the adhered cement mortar, which is not a common recycling procedure.

The basic shear strength parameters in these studies varied within the following limits:

– shear span-to-depth ratio – from 2.5 to 4.2

– concrete cylinder compressive strength – from 23 to 50 MPa

– longitudinal reinforcement ratio – from 0.53% to 4.0%

– cross-section width – from 150 to 300 mm

– cross-section height – from 230 to 550 mm

In order to take into account the effect of different shape and size of samples tested for compressive strength empirical conversion factors were used. For comparison with test shear strengths, the following codes were selected: EN 1992-1-1 [43], ACI 318M-14 [44] and CAN/CSA A23.3-04 [45]. The latter was chosen because this design procedure was derived from the Modified Compression Field Theory (MCFT) which is a comprehensive model for the response of diagonally cracked concrete subject to in-plane shear and normal stresses [46]. The predicted shear strengths were calculated on the basis of the measured concrete and steel properties and this comparison serves as the assessment of the prediction model quality and does not include safety factors, neither for loads nor for materials.

The results for RAC100, RAC50 and NAC beams are presented in Tables 6, 7 and 8, respectively. The average value of the $V_{u,test}/V_{u,code}$ ratio is similar for RAC100 and RAC50 beams, **5% to 6% lower than for NAC beams, regardless of the analyzed code.** Similar values for NAC beams in comparison to EN 1992-1-1 and ACI 318-05 were obtained also by Pérez et al. [47] on a large database with 1148 test data. The coefficient of variation (COV) of RAC100 beams is very similar to that of the NAC beams (0.22–0.35, depending on the code), while for RAC50 beams COV is lower, ranging from 0.14 to 0.22 and indicating that in this case a scatter of results is less pronounced. **From the data presented in Tables 6, 7 and 8, it is obvious that own experimental data differ significantly from the data of other researchers. The most important factor for the high $V_{u,test}/V_{u,code}$ ratios in own test results is the longitudinal reinforcement ratio (ρ) equal to approximately 4% (Figure 1), which can be considered as high. Some of the highest longitudinal reinforcement ratios in the RAC database were 2.98% [38], 2.71% [14] and 2.46% [7]. This value of ρ was chosen for two reasons: (1) in order to prevent a flexural failure of beams and (2) to extend the data from the database of RAC beams tested in shear. Under these circumstances, obviously, the 'dowel effect' is responsible for carrying a higher portion of the shear force in the total shear mechanism than it is expected and covered by codes. For example, the limit value of the longitudinal reinforcement ratio that can be taken into account in calculation of concrete shear capacity ($V_{Rd,c}$) according to EN 1992-1-1 is 2%. Although there is no example of such a high value of ρ in the database of RAC beams tested in shear, it can be found in the database of NAC beams** [48]**. Compared with $V_{u,test}/V_{u,ACI}$ ratios for NAC beams in this**

database with ρ values close to 4%, the ratios of 1.39 and 1.57 which were obtained in the research presented here for RAC50 and RAC100, respectively, seem not to deviate from the general picture.

For RAC beams with shear reinforcement, the results of 22 beams were evaluated, 14 RAC and 8 NAC corresponding beams. Beside own results, test data were taken from [8,11,12] with the same limitations as for beams without shear reinforcement. The absorption of applied RCA ranged from 3.3% to 4.8% while the oven-dry density varied from 2300 kg/m^3 to 2430 kg/m^3. In these studies the relevant shear strength parameters varied within the following limits:

– shear span-to-depth ratio – from 2.6 to 4.2

– concrete cylinder compressive strength – from 30 to 44 MPa

– longitudinal reinforcement ratio – from 2.5% to 4.0%

– shear reinforcement ratio – from 0.12% to 0.22%

– cross-section width –200 mm in all studies

– cross-section height – from 300 to 385 mm

The results for RAC100, RAC50 and NAC beams are presented in Tables 9, 10 and 11, respectively. The predictions are very conservative with relatively high coefficients of variation for all types of concrete, especially according to EN 1992-1-1 [43]. **Somewhat higher coefficients of variation for the group of beams with transverse reinforcement compared with those without it can be caused by a relatively small database of only 2 to 4 experimental programmes (Tables 9, 10 and 11). The best prediction of the ultimate shear load was obtained by ACI 318M-14 specifications** [44] **with the lowest coefficient of variation equal to 0.27.**

However, the reliability of the analyzed codes in the prediction of the ultimate shear load for beams without shear reinforcement is slightly smaller in case of RAC beams than for those made with NAC. This can be explained as a contribution of RCA. However, the existing codes predictions for NAC beams can be also used for RAC beams as the $V_{u,test}/V_{u,code}$ ratios have values greater than 1.0.

Generally, codes give more conservative predictions for beams with transverse reinforcement then without it both for NAC and RAC beams. Almost the same $V_{u,test}/V_{u,code}$ ratios for RAC and NAC beams according to ACI 318M-14 provisions [44] **are expected as the contribution of concrete in shear resistance is relatively low in this case and thus the influence of RCA is reduced compared**

with the beams without shear reinforcement. The same conclusion stands in case CAN/CSA A23.3-04 [45] **or EN 1992-1-1** [43] **specifications were used, having in mind that differences of 5-6% are rather small taking into account relatively large coefficient of variation.**

5. Conclusions

A total of 95 concrete samples as well as 9 full-scale beams were tested until failure. Based on the presented data of own experimental investigation and comparison of an available test database with selected codes' predictions, the following conclusion were drawn.

1. Concrete with 50% and 100% of coarse RCA had a lower modulus of elasticity (up to 13%) and similar tensile strength as the corresponding NAC with similar compressive strength.
2. Differences between service load deflections of the beams with 0%, 50% and 100% of RCA with the same amount of shear and longitudinal reinforcement were within a 10% difference.
3. First shear cracks in NAC and RAC beams without shear reinforcement were formed at the similar load level. First shear cracks in all three beams were actually formed before they were observed by naked eye but that moment should be precisely determined by analysis on micro scale level.
4. Shear failure modes of RAC beams without shear reinforcement did not differ from the failure mode of NAC beam, but slightly different angle and shape of the shear crack was observed in RAC50 and RAC100 beams compared with the NAC beam without shear reinforcement.
5. Shear failure modes of beams with shear reinforcement did not depend on the amount of RCA; Crack patterns were also similar although in RAC beams, the formation of series of thinner and shorter cracks between dominant inclined cracks was observed. Some differences in crack angles were noticed between RAC and NAC beams without shear reinforcement.
6. Strains in concrete and strains in longitudinal reinforcement in beams with 0%, 50% and 100% of RCA, with the same amount of shear and longitudinal reinforcement can be assessed as equal.
7. Almost all stirrups in all tested beams reached at least yielding strain at the failure load. This points to a similar contribution of the shear reinforcement and a similar contribution of the

aggregate interlock in the shear transfer mechanism of NAC and RAC beams with shear reinforcement.

8. Differences in normalized shear strengths of beams with 0%, 50% and 100% of RCA and the same amount of shear reinforcement were limited to 5%. While the shear strength of NAC and RAC100 beams without shear reinforcement was practically equal, RAC50 beam showed almost 15% lower shear strength than NAC-1 beam.
9. Overall, the shear behaviour and the shear strength of beams with the same amount of shear reinforcement were very similar, regardless of the amount of RCA.
10. The shear strength of RAC50 and RAC100 beams with and without shear reinforcement was conservatively predicted by the analyzed codes with similar reliability as for the corresponding NAC beams shear strength. At this state-of-knowledge, the application of the analyzed codes' provisions for NAC beams shear strength can be recommended both for the RAC50 and the RAC100 beams.

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Table 1
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Table 1. Natural aggregate and recycled aggregate properties

Type of aggregate	Grain size [mm]	Volumetric coefficient [%]	Crushing resistance (cylinder) [%]	Fines content < 0.063 [%]	SSD[2] density [kg/m^3]	Bulk density [kg/m^3]	Loose bulk density [kg/m^3]	Water absorption [%]		
								10 min	30 min	24h
RCA	4/8	29	23	0.38	2405	1275	1132	4.1	4.1	4.6
	8/16	20.7	26.1	0.38	2456	1398	1260	3.5	4	3.7
	16/31.5	28.6	32.7	0	2482	1694	1521	3.4	3.7	3.8
NA	4/8	27.4	21	0.23	2593	1532	1406	0.8	0.8	0.9
	8/16	27.1	24	0.08	2595	1587	1463	0.6	0.6	0.8
	16/31.5	29.7	28.9	0.12	2601	1941	1814	0.4	0.5	0.5
Quality requirement for NA		≥18	< 30	< 1	2000-3000	-	-	-	-	≤1.5
Standard		[20]	[21]	[22]	[23]	[24]	[25]		[23]	

[1] Serbian standards for aggregates, provided in references
[2] SSD- Saturated Surface Dry

Table 2
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Table 2. Composition of concrete mixtures in [kg/m^3]

Concrete mix ID	Cement	Water		w/c[1]	Natural aggregate				Recycled aggregate		
		"free"	added		Sand	Coarse aggregate			Coarse aggregate		
					0/4	4/8	8/16	16/31.5	4/8	8/16	16/31.5
NAC	302	180	-	0.596	619	310	310	583	-	-	-
RAC50	302	180	21	0.596	618	132	177	265	132	177	265
RAC100	312	181	40	0.580	597	-	-	-	204	391	511

[1] "free" water-to-cement ratio

Table 3
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Table 3. Properties of fresh and hardened concrete

Concrete mix ID	Properties of fresh concrete			Properties of hardened concrete					
	Density [kg/m^3]	Slump [mm]		Density [kg/m^3]	$f_{c,cube}$[2] [MPa]		E_c[3] [GPa]	$f_{ct,sp}$[4] [MPa]	$f_{ct,fl}$[5] [MPa]
		imme-diately	after 30 min.		28 - day	test date[1]	28 days	28 days	28 days
NAC	2401	118	63	2384	38.5	41.8	28.0	3.0	7.0
RAC50	2356	110	65	2345	41.8	46.3	24.5	3.2	7.3
RAC100	2331	127	63	2320	43.1	42.4	25.1	3.1	6.6

[1]26 days old sample for NAC, RAC50 and RAC100
[2]cube, 150 mm; [3]cylinder, Ø150/300 mm; [4]cylinder, Ø150/150 mm; [5]prism, 120/120/360 mm

Table 4

Table 4. Properties of steel bars used as a longitudinal reinforcement

Diameter	Yield stress [MPa]	Tensile stress [MPa]	Elongation at break [%]
Ø10	638	706	5.8
Ø16	580	648	8.0
Ø18	547	619	7.6
Ø22	555	616	9.3

Table 5

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Table 5. Summary of test results

ID of beam	ρ_1 [%]	$2P_{cr,fl}$ [kN]	$2P_{cr,s}$ [kN]	$2P_u$ [kN]	$2P_u$/ $2P_{cr,s}$	δ_s [mm]	$v_{cr,s}/f_c^{1/3}$	$v_u/f_c^{1/3}$
NAC-1	0	37	160	212.5	1.33	5.10	0.114	0.152
RAC50-1	0	40	160	183.5	1.15	4.45	0.110	0.127
RAC100-1	0	50	160	209.5	1.31	4.66	0.114	0.149
NAC-2	0.14	35	160	281.3	1.76	6.87	0.114	0.201
RAC50-2	0.14	40	160	283.9	1.77	7.62	0.110	0.196
RAC100-2	0.14	30	160	269.9	1.69	6.67	0.114	0.192
NAC-3	0.19	30	180	319.8	1.78	7.64	0.129	0.228
RAC50-3	0.19	30	180	313.7	1.74	7.41	0.124	0.216
RAC100-3	0.19	30	180	326.8	1.82	8.01	0.128	0.232

Table 6
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Table 6. Comparison of test results to code provisions for RAC100 beams without shear reinforcement

Author	TEST V_u (kN)	EC2[1] V_u (kN)	ACI 318 V_u (kN)	CSA V_u (kN)	$V_{u,test}/V_{u,EC2}$	$V_{u,test}/V_{u,ACI}$	$V_{u,test}/V_{u,CSA}$
Han et al. [2.1]	55.1	50.0	43.6	43.9	1.10	1.26	1.26
	50.9	50.3	44.1	39.1	1.01	1.15	1.30
Etxeberria et al. [2.3]	84.0	85.0	65	90.8	0.99	1.29	0.93
Fathifazl et al. [2.4]	105.6	91.7	72.7	75.7	1.15	1.45	1.40
	122.6	66.6	47.9	44.1	1.84	2.56	2.78
	111.7	105.9	90.8	101.1	1.05	1.23	1.10
	119.6	122.9	113.4	122.3	0.97	1.05	0.98
Choi et al. [2.6]	84.8	75.0	58.2	73.6	1.13	1.46	1.15
	57.8	75.0	58.2	76.9	0.77	0.99	0.75
	59.8	51.8	58.2	50.1	1.16	1.03	1.19
	70.1	60.1	58.2	58.9	1.17	1.20	1.19
Arezoumandi et al. [2.7]	114.8	124.1	111.7	125.3	0.93	1.03	0.92
	143.2	137.2	104.8	125.7	1.04	1.37	1.14
	131.4	137.2	104.8	145.5	0.96	1.25	0.90
	113.0	129.5	119.1	134.6	0.87	0.95	0.84
	124.1	143.2	111.7	142.3	0.87	1.11	0.87
	140.3	143.2	111.7	151.6	0.98	1.26	0.93
Knaack & Kurama [2.8]	36.4	40.8	32.8	32.2	0.89	1.11	1.13
	38.0	40.8	32.8	31.4	0.93	1.16	1.21
	39.9	38.8	30.5	28.3	1.03	1.31	1.41
	36.1	38.8	30.5	30.0	0.93	1.18	1.20
Own	104.8	66.7	46.9	59.0	1.57	2.23	1.77
				Average	1.06	1.30	1.20
				Standard deviation	0.24	0.38	0.42
				COV[2]	0.22	0.29	0.35

[1] EC2 is an abbreviation for EN 1992-1-1
[2] Coefficient of variation

Table 7
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Table 7. Comparison of test results to code provisions for RAC50 beams without shear reinforcement

Author	TEST V_u (kN)	EC2[1] V_u (kN)	ACI 318 V_u (kN)	CSA V_u (kN)	$V_{u,test}/V_{u,EC2}$	$V_{u,test}/V_{u,ACI}$	$V_{u,test}/V_{u,CSA}$
Fonteboa and Abella [2.2]	90.6	85.0	64.9	88.1	1.07	1.40	1.03
Etxeberria et al. [2.3]	89.0	86.1	66.2	90.6	1.03	1.34	0.98
Fathifazl et al. [2.4]	103.9	81.7	67.8	71.5	1.27	1.53	1.45
	83.2	86.7	66.9	78.2	0.96	1.24	1.06
	89.3	63.0	44.1	47.5	1.42	2.03	1.88
	99.5	100.2	83.6	97.9	0.99	1.19	1.02
	104.6	116.3	104.4	119.1	0.90	1.00	0.88
Choi et al. [2.6]	87.9	76.6	60.1	74.8	1.15	1.46	1.17
	71.6	76.6	60.1	72.7	0.93	1.19	0.98
	57.8	52.9	60.1	52.8	1.09	0.96	1.10
	67.1	61.4	60.1	62.3	1.09	1.12	1.08
Arezoumandi et al. [2.9]	117.5	126.9	115.6	128.2	0.93	1.02	0.92
	151.3	140.3	108.4	126.9	1.08	1.40	1.19
	171.8	140.3	108.4	136.0	1.22	1.59	1.26
	111.7	131.2	121.5	138.0	0.85	0.92	0.81
	148.6	145.1	114.0	134.5	1.02	1.30	1.10
	168.7	145.1	114.0	144.1	1.16	1.48	1.17
Knaack and Kurama [2.8]	44.0	41.5	33.7	29.4	1.06	1.31	1.49
	39.1	41.5	33.7	31.6	0.94	1.16	1.24
	43.7	40.4	32.3	28.4	1.08	1.35	1.54
	41.2	40.4	32.3	29.4	1.02	1.27	1.40
Own	91.8	66.0	46.2	61.8	1.39	1.99	1.49
				Average	1.08	1.33	1.19
				Standard deviation	0.15	0.29	0.26
				COV[2]	0.14	0.21	0.22

[1] EC2 is an abbreviation for EN 1992-1-1
[2] Coefficient of variation

Table 8
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Table 8. Comparison of test results to code provisions for NAC beams without shear reinforcement

	TEST	EC2[1]	ACI 318	CSA			
Author	V_u (kN)	V_u (kN)	V_u (kN)	V_u (kN)	$V_{u,test}/V_{u,EC2}$	$V_{u,test}/V_{u,ACI}$	$V_{u,test}/V_{u,CSA}$
Gonzalez and Abella [2.2]	88.9	85.3	65.3	89.3	1.04	1.36	1.00
Etxeberria et al. [2.3]	100.5	86.5	66.7	86.8	1.16	1.51	1.16
Fathifazl et al. [2.4]	92.8	79.8	65.4	73.0	1.16	1.42	1.27
	150.0	76.7	61.6	53.0	1.96	2.43	2.83
Choi et al. [2.6]	90.7	77.2	60.8	74.8	1.17	1.49	1.21
	71.1	77.2	60.8	73.9	0.92	1.17	0.96
	66.2	53.3	60.8	49.2	1.24	1.09	1.35
	72.0	61.9	60.8	60.8	1.16	1.18	1.19
Arezoumandi et al. [2.7]	121.2	133.4	124.6	136.2	0.91	0.97	0.89
	143.2	147.5	116.8	140.1	0.97	1.23	1.02
	173.5	147.5	116.8	146.0	1.18	1.49	1.19
	129.9	129.6	119.3	126.1	1.00	1.09	1.03
	167.0	143.3	111.8	125.1	1.17	1.49	1.33
	170.8	143.3	111.8	140.7	1.19	1.53	1.21
Knaack and Kurama [2.8]	31.1	37.7	29.1	31.2	0.83	1.07	1.00
	36.9	37.7	29.1	28.3	0.98	1.27	1.30
	40.4	43.5	36.2	33.3	0.93	1.12	1.21
	42.3	43.5	36.2	32.4	0.97	1.17	1.31
Own	106.3	64.2	44.3	55.4	1.65	2.40	1.92
				Average	1.14	1.39	1.28
				Standard deviation	0.27	0.40	0.43
				COV[2]	0.24	0.29	0.34

[1] EC2 is an abbreviation for EN 1992-1-1
[2] Coefficient of variation

Table 9
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Table 9. Comparison of test results to code provisions for RAC100 beams with shear reinforcement

Author	TEST V_u (kN)	EC2[1] V_u (kN)	ACI 318 V_u (kN)	CSA V_u (kN)	$V_{u,test}/V_{u,EC2}$	$V_{u,test}/V_{u,ACI}$	$V_{u,test}/V_{u,CSA}$
Etxeberria et al. [2.3]	189.5	161.3	136.7	153.3	1.17	1.39	1.24
	163.0	123.4	119.8	139.5	1.32	1.36	1.17
	168.0	156.1	134.3	157.0	1.08	1.25	1.07
Own	135.0	66.7	66.9	78.3	2.02	2.02	1.72
	163.4	66.7	73.5	80.3	2.45	2.22	2.03
				Average	1.61	1.65	1.45
				Standard deviation	0.60	0.44	0.41
				COV[2]	0.37	0.27	0.29

[1] EC2 is an abbreviation for EN 1992-1-1
[2] Coefficient of variation

Table 10
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Table 10. Comparison of test results to code provisions for RAC50 beams with shear reinforcement

Author	TEST V_u (kN)	EC2[1] V_u (kN)	ACI 318 V_u (kN)	CSA V_u (kN)	$V_{u,test}/V_{u,EC2}$	$V_{u,test}/V_{u,ACI}$	$V_{u,test}/V_{u,CSA}$
Fonteboa and Abella [2.2]	164.3	84.7	100.3	113.5	1.94	1.64	1.45
	177.0	113.4	116.8	131.0	1.56	1.52	1.35
	233.6	148.3	131.5	134.8	1.58	1.78	1.73
Etxeberria et al. [2.3]	220.0	161.3	137.9	145.9	1.36	1.59	1.51
	176.0	123.4	121.1	136.9	1.43	1.45	1.29
	164.0	87.4	105.1	119.5	1.88	1.56	1.37
Fathifazl et al. [2.5]	172.0	183.4	148.6	175.1	0.94	1.16	0.98
Own	142.0	66.0	66.1	75.8	2.15	2.15	1.87
	156.9	66.0	72.8	81.0	2.38	2.16	1.94
				Average	1.69	1.67	1.50
				Standard deviation	0.44	0.32	0.30
				COV[2]	0.26	0.19	0.20

[1] EC2 is an abbreviation for EN 1992-1-1
[2] Coefficient of variation

Table 11. Comparison of test results to code provisions for NAC beams with shear reinforcement

Author	TEST V_u (kN)	EC2[1] V_u (kN)	ACI 318 V_u (kN)	CSA V_u (kN)	$V_{u,test}/V_{u,EC2}$	$V_{u,test}/V_{u,ACI}$	$V_{u,test}/V_{u,CSA}$
Fonteboa and Abella [2.2]	128.0	84.6	100.2	124.5	1.51	1.28	1.03
	150.8	113.4	114.8	136.9	1.33	1.31	1.10
	190.3	148.3	129.2	144.1	1.28	1.47	1.32
Etxeberria et al. [2.3]	213.0	161.3	138.4	148.2	1.32	1.54	1.44
	177.0	123.4	121.5	137.0	1.43	1.46	1.29
	187.5	87.4	105.5	113.6	2.15	1.78	1.65
Own	140.7	64.2	64.3	74.1	2.19	2.19	1.90
	159.9	64.2	70.9	78.5	2.49	2.25	2.04
				Average	1.71	1.66	1.47
				Standard deviation	0.48	0.38	0.36
				COV[2]	0.28	0.23	0.25

[1] EC2 is an abbreviation for EN 1992-1-1
[2] Coefficient of variation

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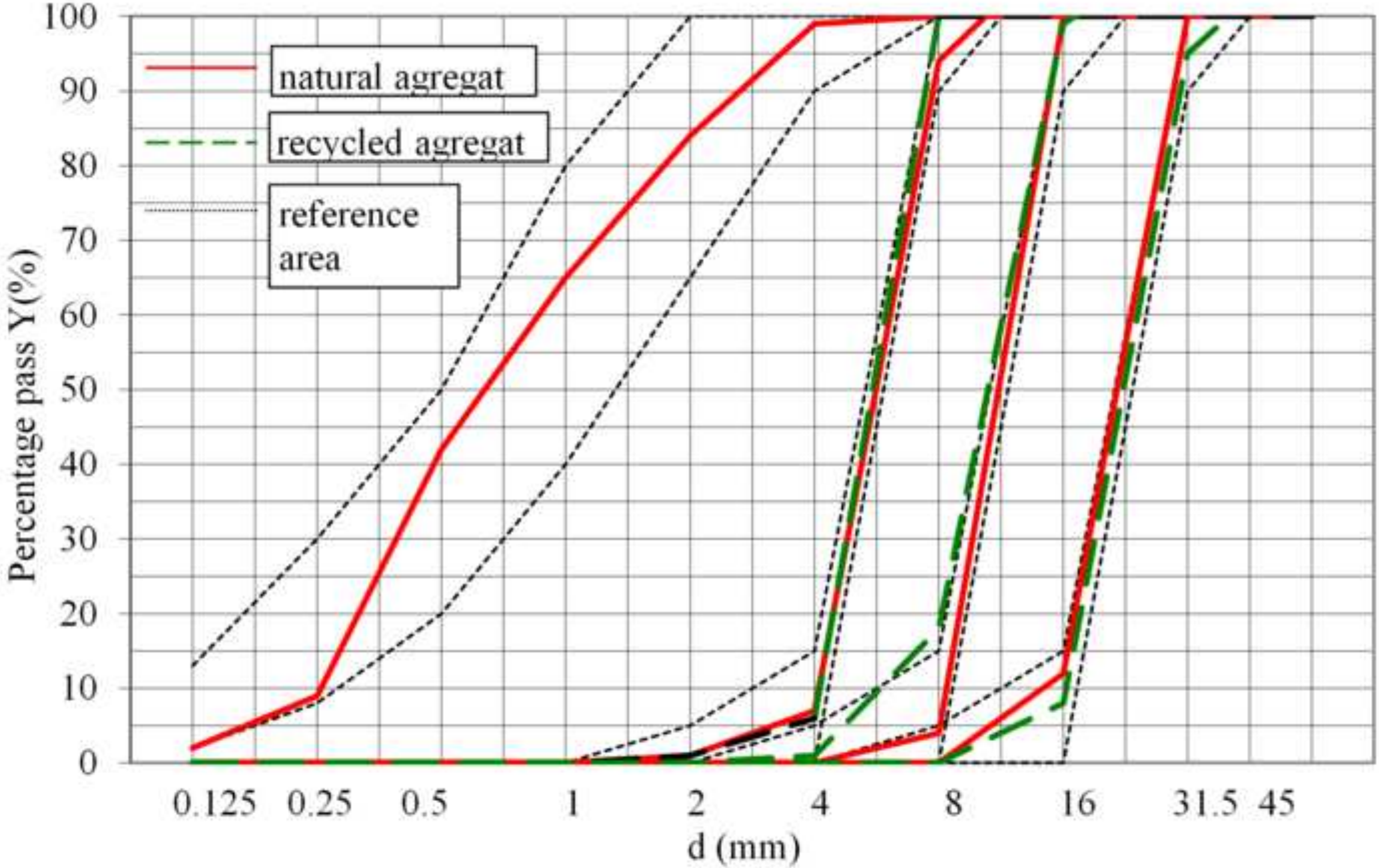

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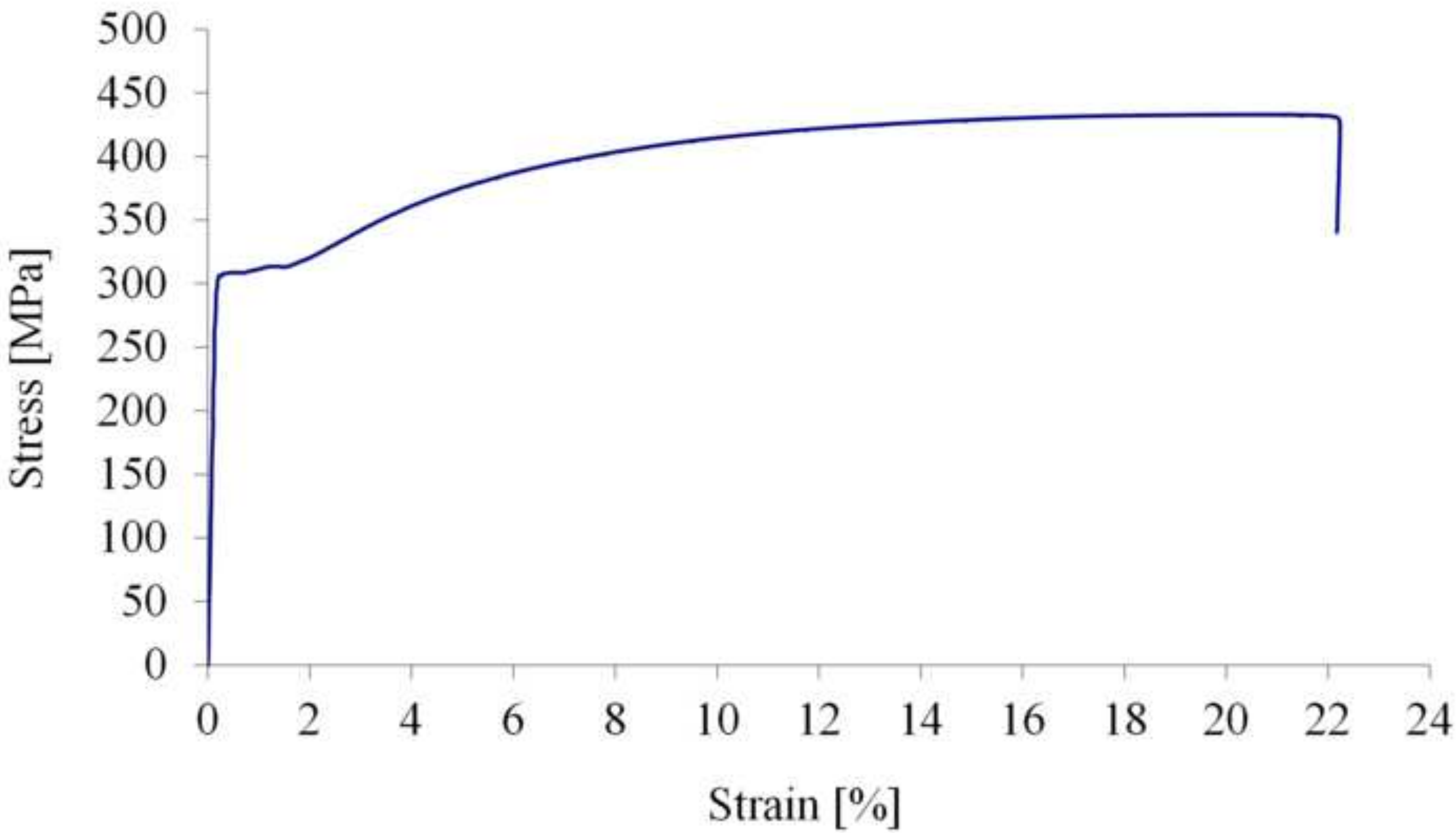

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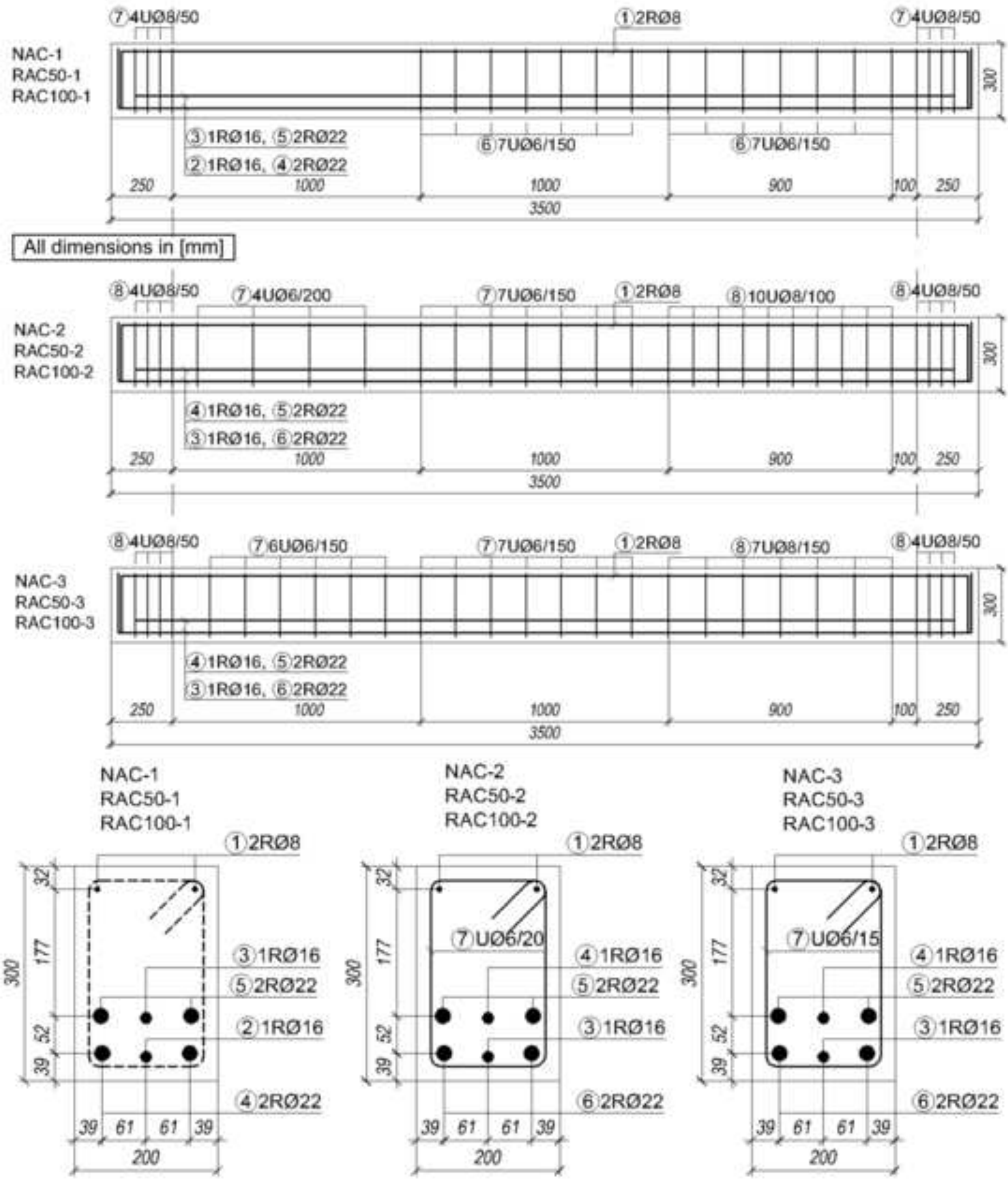

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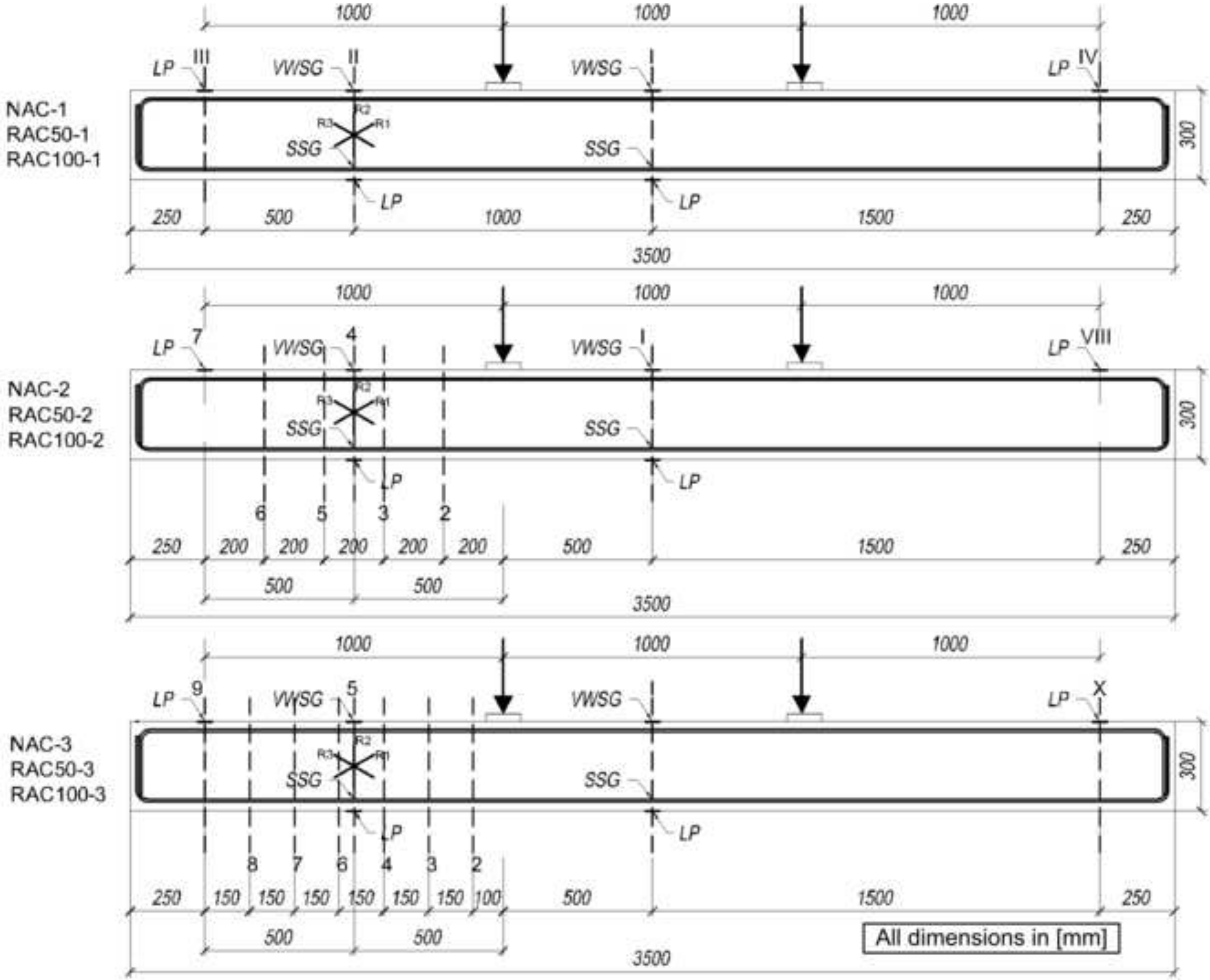

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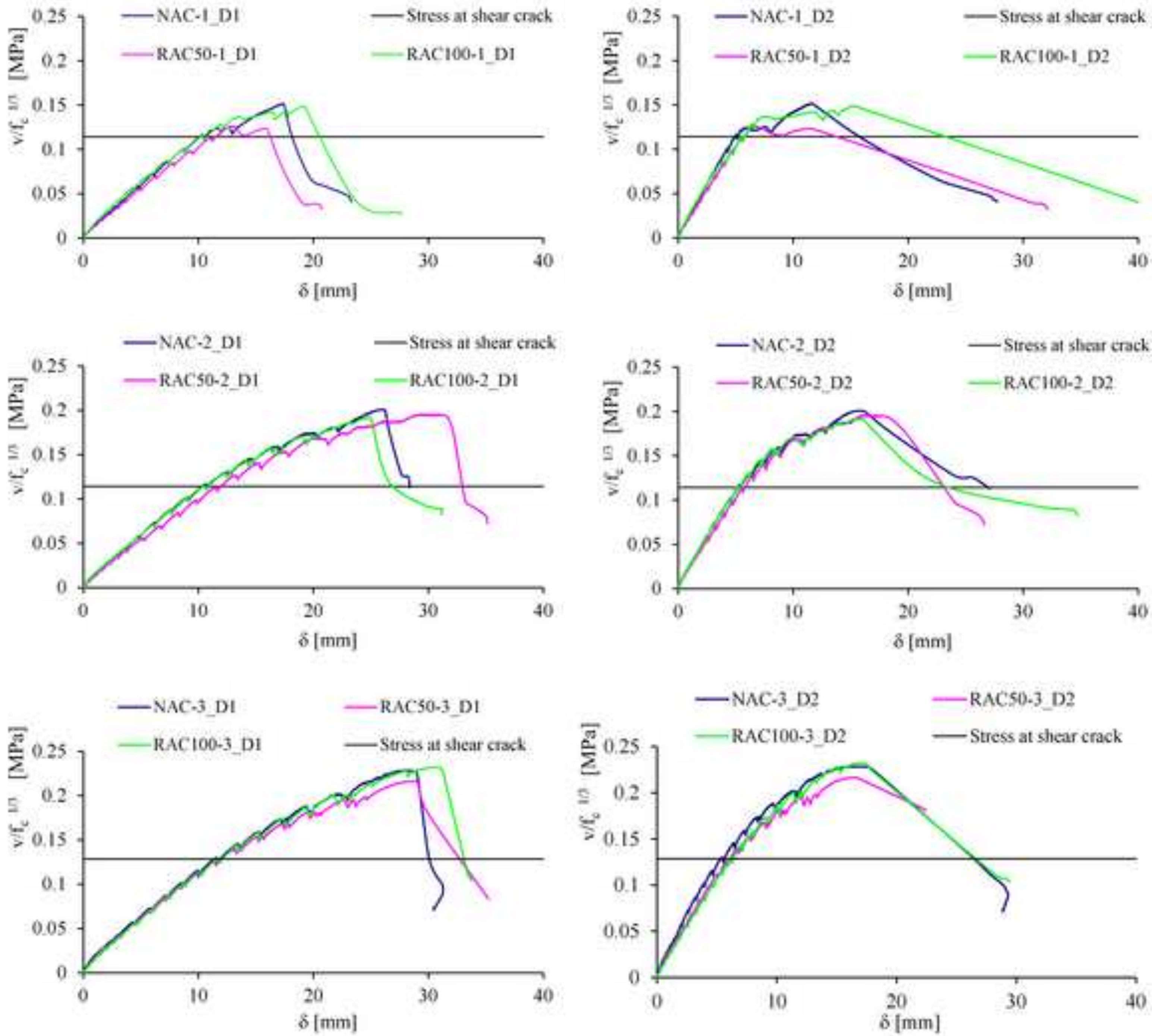

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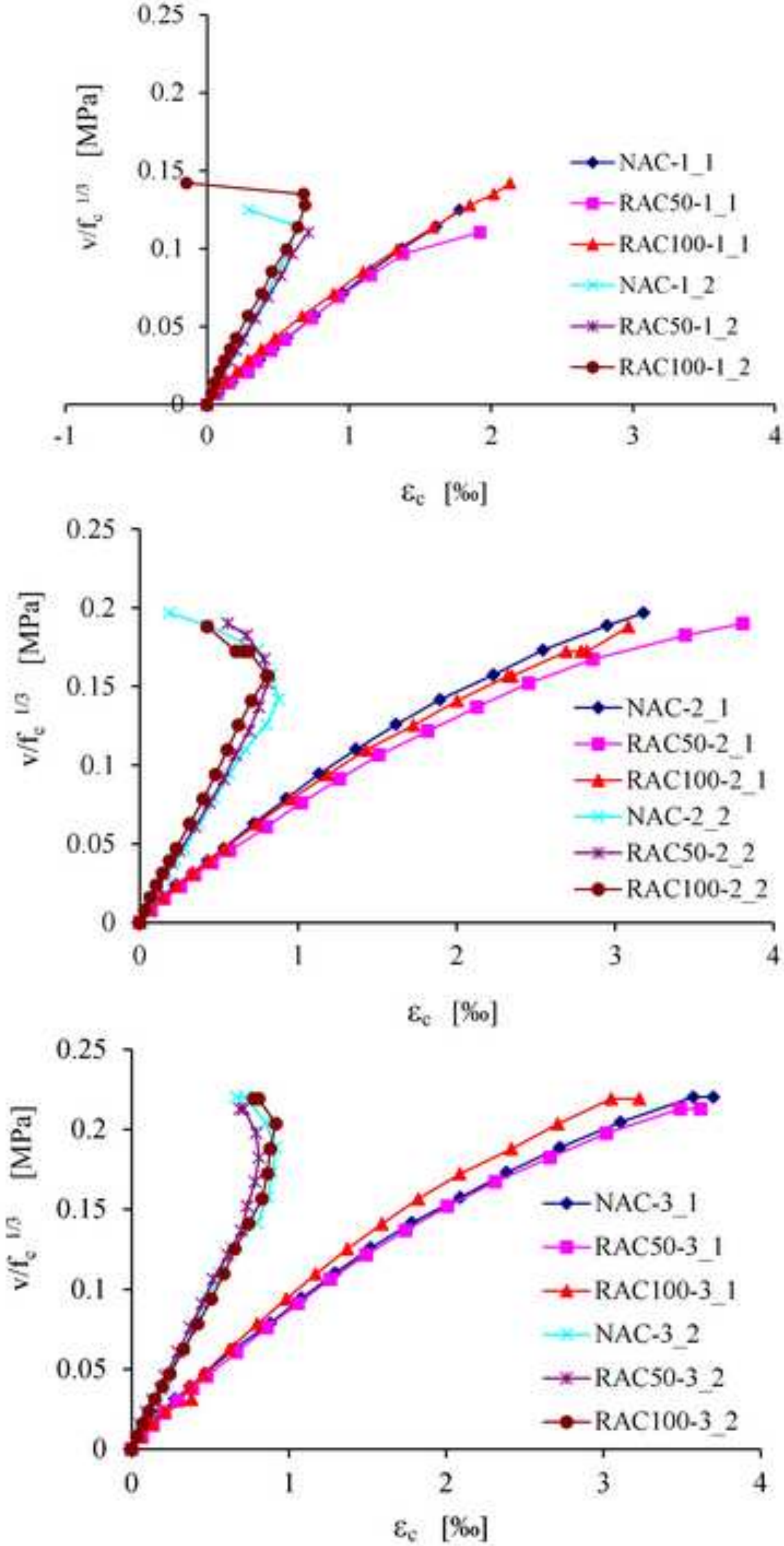

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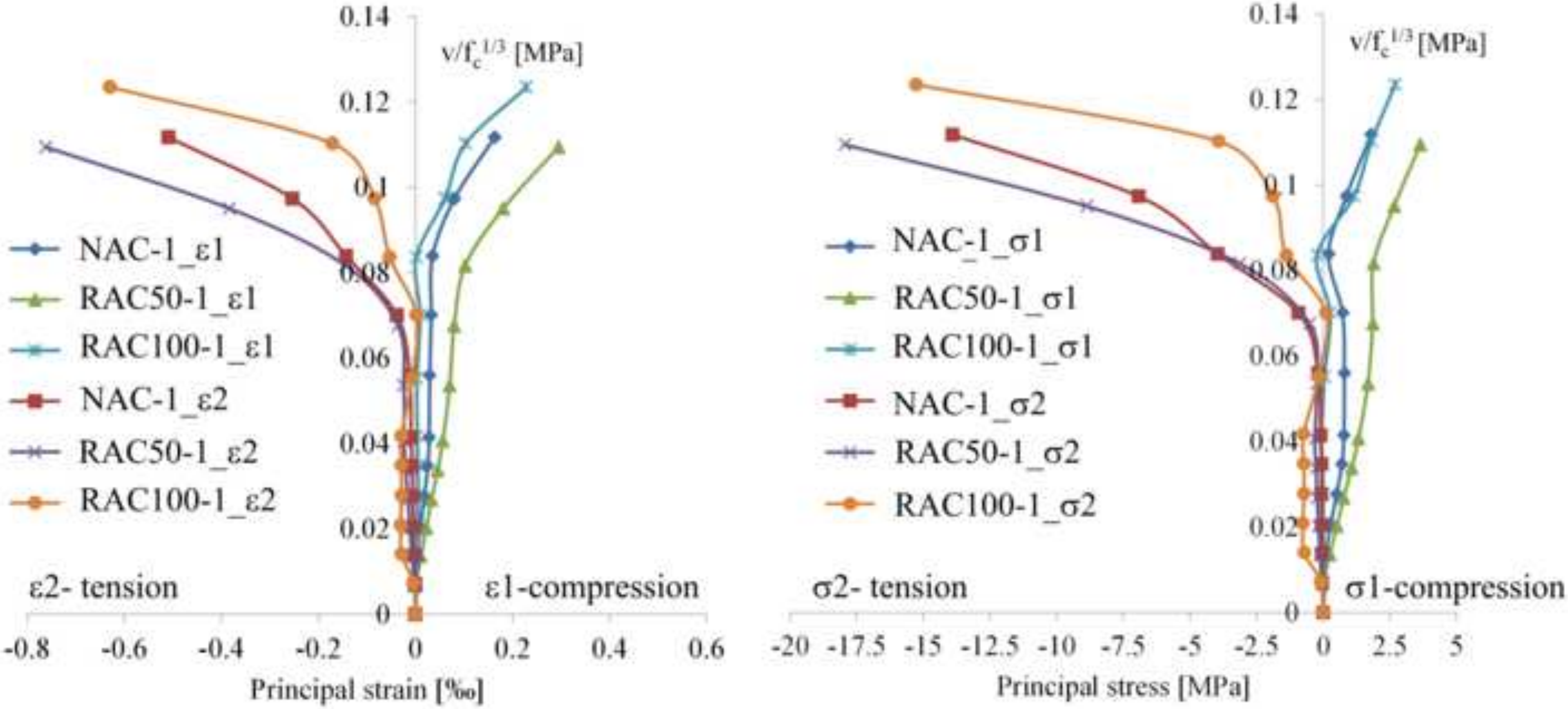

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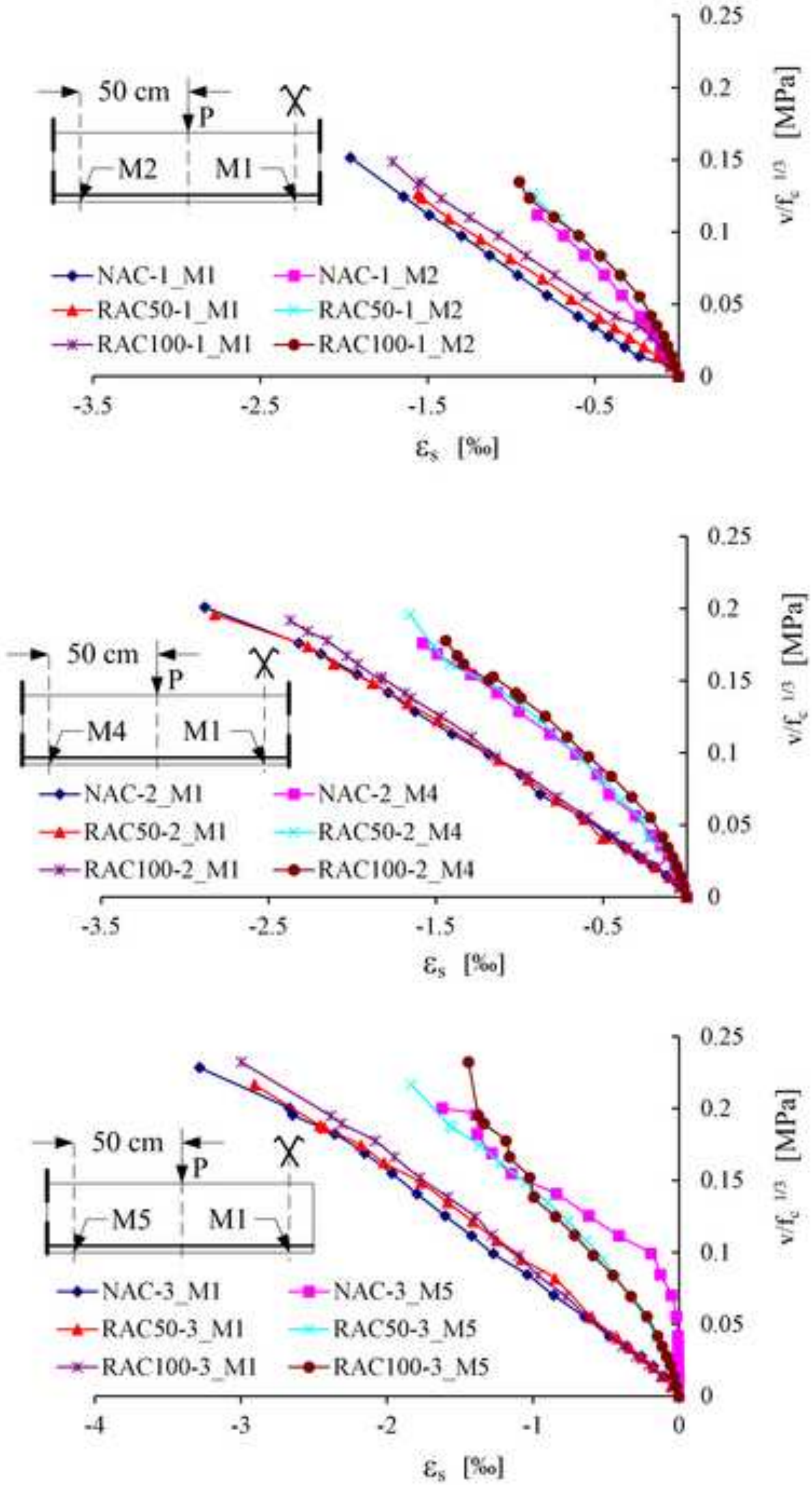

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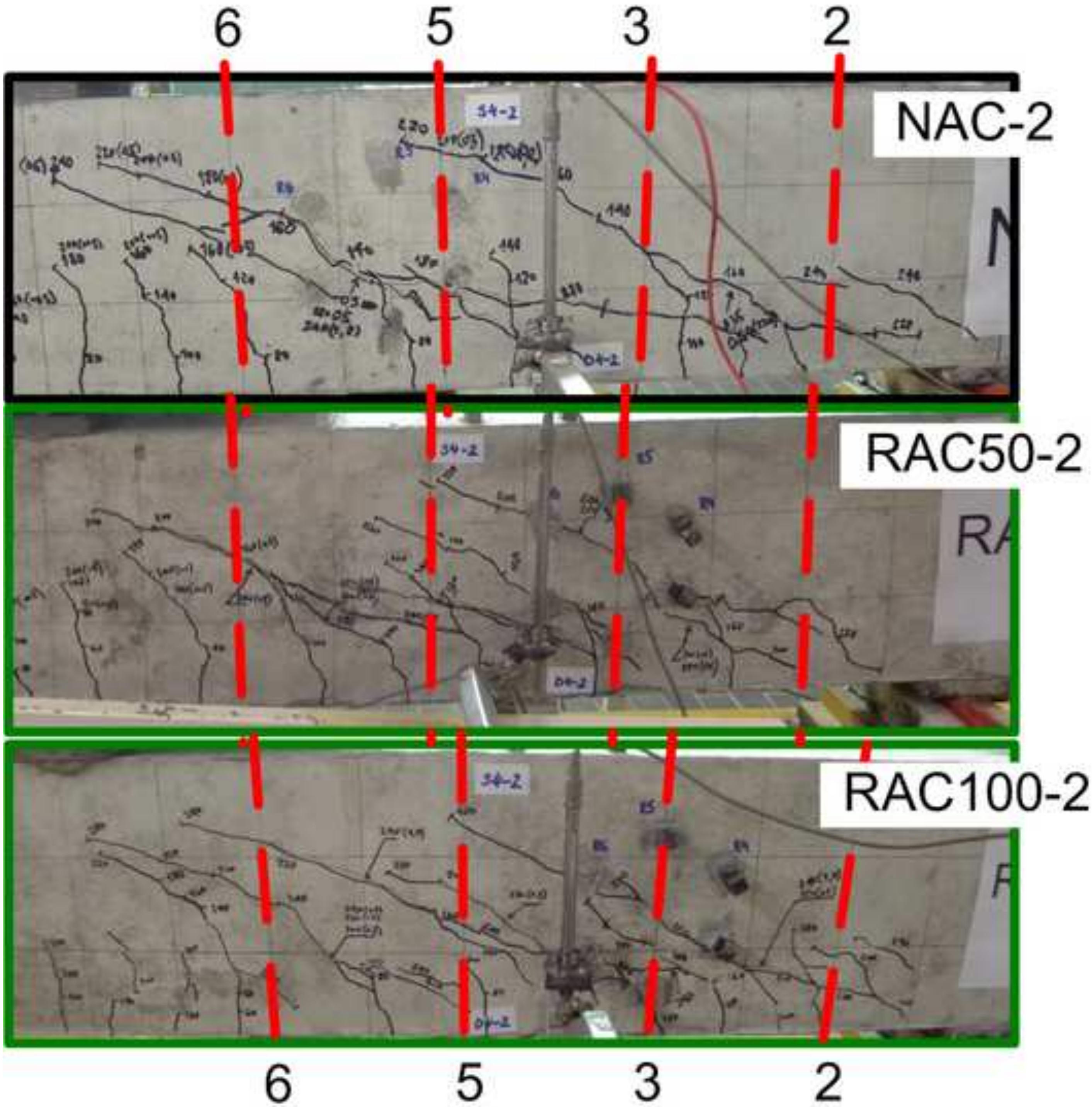

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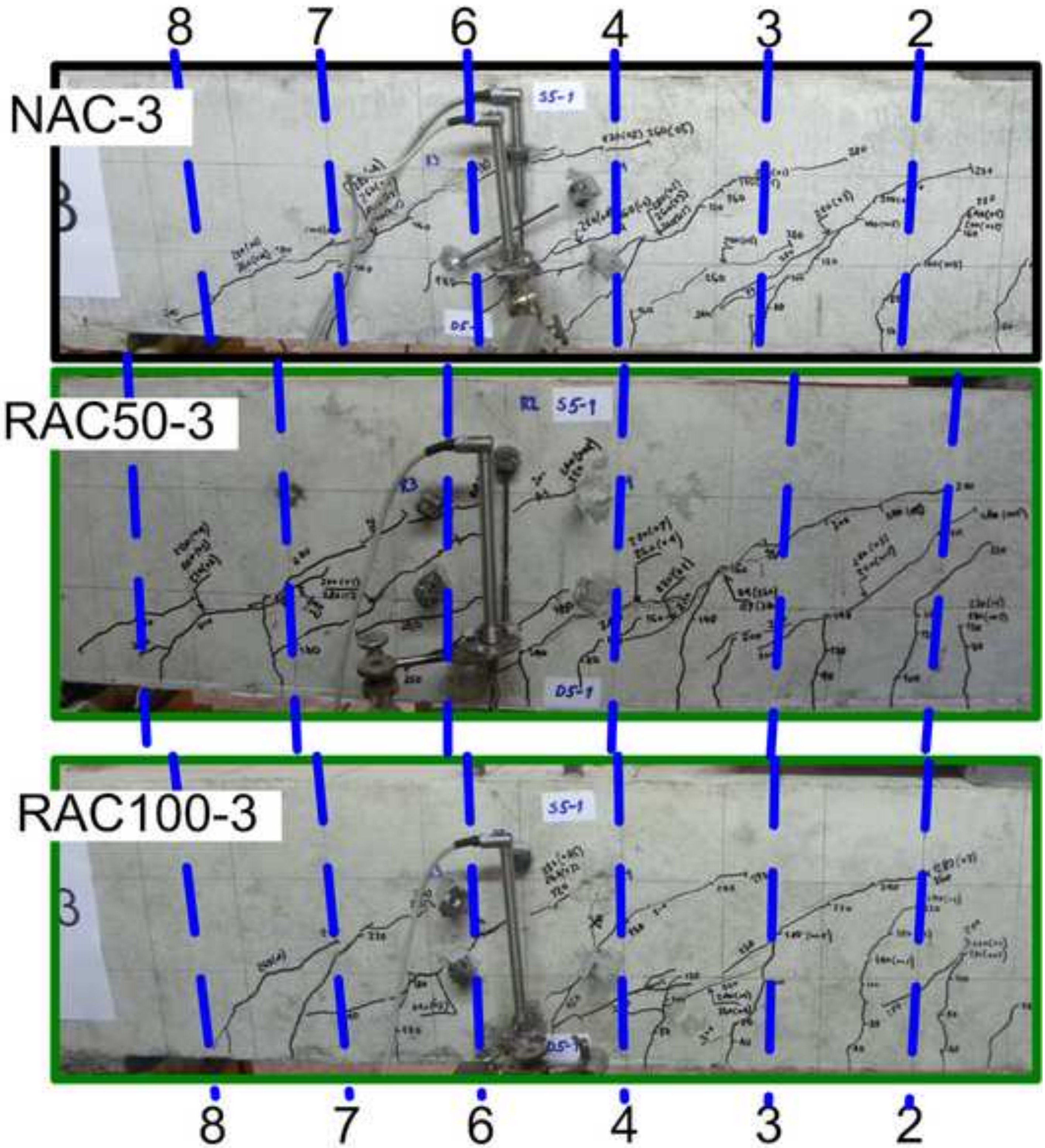

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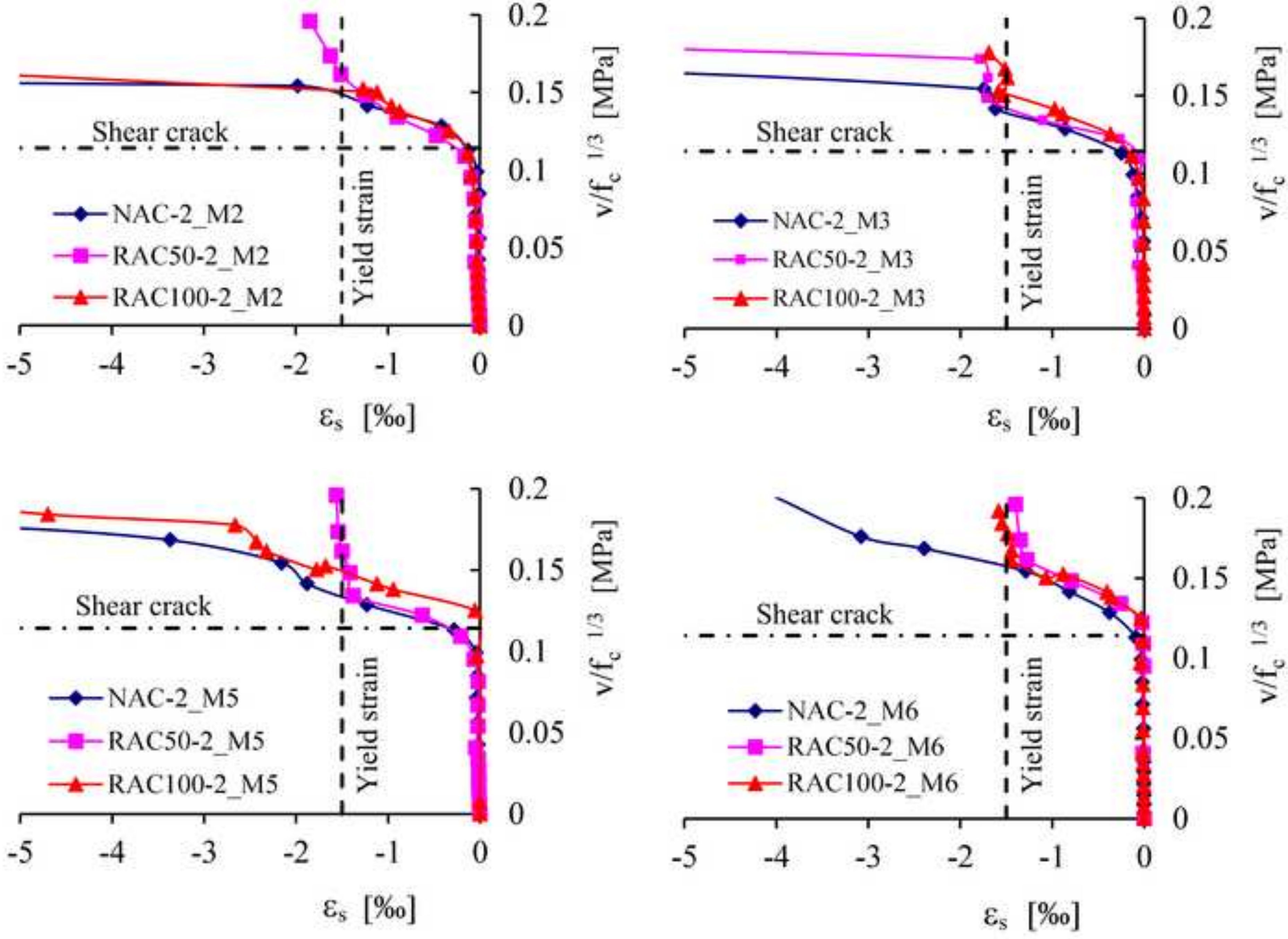

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